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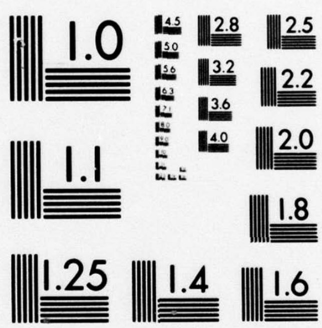

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Overlay of Large, Placed Quarrystone and Boulders to Increase Riprap Stability

by

Bruce L. McCartney and John P. Ahrens

TECHNICAL PAPER NO. 76-19
DECEMBER 1976

U.S. ARMY, CORPS OF ENGINEERS
COASTAL ENGINEERING RESEARCH CENTER
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Please make the following corrections:

Page 30:

$$W_{50} = \frac{w_r \; H^3}{N_s^3 \quad (S_r - 1)^3}$$

Page 33:

$$W_{50} = H^3 \, w_r \,/\, [\, N_s{}^3 \, (S_r - 1)^3]$$

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report describes the wave tank tests and field performance of a single layer of large armor stone used as a protective overlay on under-designed riprap. The resistance of the overlay to wave attack was determined by small-scale model and prototype-scale wave tank tests at CERC.

A stone overlay concept was successfully used to repair a damaged riprap revetment on Oahe Reservoir, South Dakota (App. A). Design information and (continued)

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an example problem (App. B) are presented to demonstrate application of the overlay concept to both revetment repair and original construction.

PREFACE

This report is published to assist engineers in the planning, design, and construction of riprap-type revetments exposed to wave attack. The work was carried out under the coastal construction research program of the U.S. Army Coastal Engineering Research Center (CERC).

The report was prepared by Bruce L. McCartney, formerly of CERC, and John P. Ahrens (who directed the tests), Coastal Structures Branch under the general supervision of Dr. Robert M. Sorensen, Chief, Coastal Structures Branch, Research Division.

The authors acknowledge Mr. Leonard Kraft of the Foundations and Materials Branch, U.S. Army Engineer District, Omaha, for the photos of the riprap and overlay on the Mobridge embankment. Technical assistance was also provided by Mr. Arvid Thomsen, Chief, Economics and Social Analysis Branch, Omaha District, and Mr. Paul Wohlt, formerly of the Geology, Soils, and Material Branch, and Mr. Alfred Harrison, Chief, Hydraulics and Hydrology Section, U.S. Army Engineer Division, Missouri River. Mr. Robert Jachowski, Chief, Design Branch, Engineering Development Division, CERC, reviewed the draft of this report and his many comments greatly improved the final study.

Comments on this publication are invited.

Approved for publication in accordance with Public Law 166, 79th Congress, approved 31 July 1945, as supplemented by Public Law 172, 88th Congress, approved 7 November 1963.

John H. Cousins
JOHN H. COUSINS
Colonel, Corps of Engineers
Commander and Director

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TABLES

FIGURES

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	by	To obtain
inches	25.4	millimeters
	2.54	centimeters
square inches	6.452	square centimeters
cubic inches	16.39	cubic centimeters
feet	30.48	centimeters
	0.3048	meters
square feet	0.0929	square meters
cubic feet	0.0283	cubic meters
yards	0.9144	meters
square yards	0.836	square meters
cubic yards	0.7646	cubic meters
miles	1.6093	kilometers
square miles	259.0	hectares
acres	0.4047	hectares
foot-pounds	1.3558	newton meters
ounces	28.35	grams
pounds	453.6	grams
	0.4536	kilograms
ton, long	1.0160	metric tons
ton, short	0.9072	metric tons
degrees (angle)	0.1745	radians
Fahrenheit degrees	5/9	Celsius degrees or Kelvins[1]

[1] To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use formula: $K = (5/9)(F - 32) + 273.15$.

SYMBOLS AND DEFINITIONS

C	stone overlay weight per square foot (lb/ft^2)
C.F.	coverage fraction (dimensionless)
d	stillwater depth in wave tank (ft)
$H_{D=0}$	zero-damage wave height (ft)
$H_{D=t}$	tolerable-damage wave height (ft)
L	wavelength (ft)
N_s	conventional stability number, based on median weight, $\dfrac{H_{D=0}}{(W_{50}/w_r)^{1/3}\ (S_r - 1)}$ (dimensionless)
N_s^*	stability number, based on average weight, $\dfrac{H_{D=0}}{(W/w_r)^{1/3}\ (S_r - 1)}$ (dimensionless)
S_r	specific gravity of armor unit ($S_r = w_r/w_w$) (dimensionless)
W	average weight of riprap or stone overlay (lb)
W_{50}	median riprap or stone overlay weight (lb)
w_r	unit weight of stone (lb/ft^3)
w_w	unit weight of water (lb/ft^3)
θ	angle of the structure face relative to the horizontal (degrees)

OVERLAY OF LARGE, PLACED QUARRYSTONE AND BOULDERS TO INCREASE RIPRAP STABILITY

by

Bruce L. McCartney and *John P. Ahrens*

I. INTRODUCTION

Conventional riprap (underlayer stone) revetment design requires an armor layer thickness 1.5 to 2 times the median diameter of the armor stone. The armor layer completely buries the underlayer so the revetment derives minimal benefit from the underlayer stone stability. This report presents the results of an investigation of revetment stability to wave attack for a one-stone-diameter-thick armor overlay. The armor overlay could also be used to upgrade existing riprap or as a cost-effective initial design.

The reduced armor layer thickness concept was proposed by the U.S. Army Engineer Division, Missouri River, as a means of upgrading an existing riprap slope on the Oahe Reservoir in South Dakota. The slope was damaged by unusually high storm waves in 1967. A case study of the Oahe Reservoir riprap repair is presented in Appendix A.

II. WAVE TANK TESTS

1. General.

Wave tank testing to determine stability against wave attack for single-layer quarrystone and boulder overlays, for both 100- and 67-percent surface coverage, was conducted at the U.S. Army Coastal Engineering Research Center (CERC) in both the small and large wave tanks. The 100-percent coverage is defined as all overlay stones touching; the 67-percent coverage used two-thirds as many stones per unit area as the 100-percent coverage. These general overlayer coverage definitions were converted to a dimensionless parameter (coverage fraction) which provides a quantitative measurement of coverage. The coverage fraction is the overlay stone weight per unit area for a specific overlay percent and stone shape. Cover fraction (C.F.) is defined by:

$$C.F. = \frac{C}{\left(\frac{W}{w_r}\right)^{1/3} w_r} , \qquad (1)$$

where C is the overlay weight (pounds per square foot), w_r is the unit weight of the stone (pounds per cubic foot), and W is the average stone weight (pounds).

Fourteen model tests were conducted at a model to prototype scale of 1:10 in the small wave tank, 1.5 feet (45.7 centimeters) wide, 2 feet (61 centimeters) deep, and 72 feet (21.9 meters) long. The small wave tank tests were used to evaluate the stability of a single-layer overlay of rounded boulders and angular quarrystone. Two prototype (full-scale) tests, using a single layer of angular to blocky quarrystone, were

conducted in the large wave tank, 15 feet (4.6 meters) wide, 20 feet (6.1 meters) deep, and 635 feet (194 meters) long. The large wave tank tests were used to verify the validity of the stone overlay concept at prototype scale. Test conditions for both wave tanks are given in Table 1 (see Coastal Engineering Research Center, 1971, for a detailed description of the tanks).

2. Small-Scale Tests.

The small wave tank tests were run with a 1.5-foot water depth and a wave period of 1.16 seconds which give a depth, d, to wavelength, L, ratio of 0.24. This value (d/L = 0.24) coincides with a large number of riprap stability tests previously run at CERC (Thomsen, Wohlt, and Harrison, 1972). At the normal operating water depth of 15 feet in the large tank, d/L = 0.24 gives a wave period of 3.67 seconds, the approximate design wave period at the Mobridge, South Dakota, railroad embankment on the Oahe Reservoir.

The core of the embankment in the small tank tests was composed of packed sand with a median diameter of 0.2 millimeter. Between the core and the riprap underlayer there was a 0.5-inch (12.7 millimeters) layer of coarse filter sand with a median diameter of 1.2 millimeters. The distinctive reddish-brown color of the filter sand made exposure of the filter easy to observe. Crushed bluestone with a median diameter of 11 millimeters was used as a riprap layer. This layer was designated the riprap underlayer for the tests using an overlay. The small wave tank test setup is shown in Figure 1.

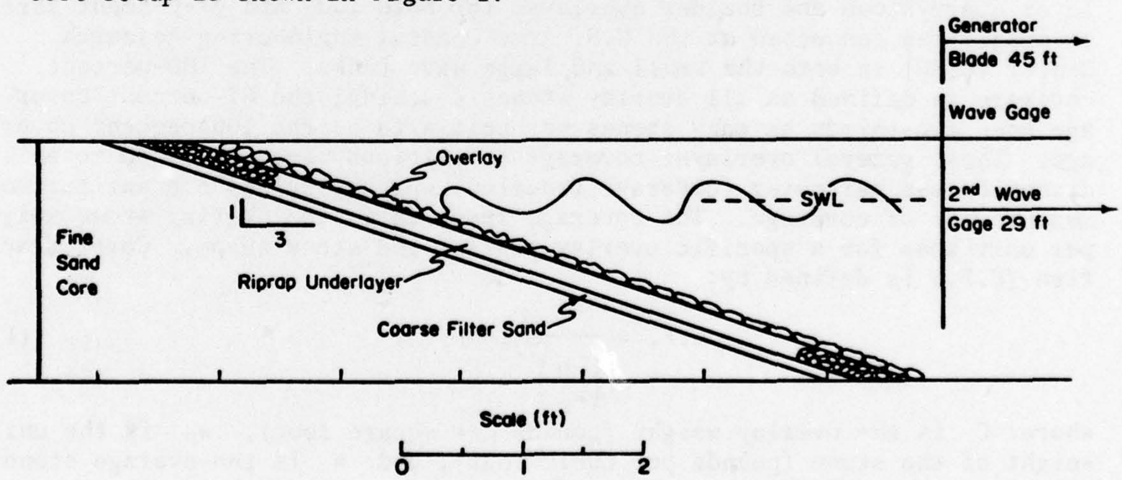


Figure 1. Details of small wave tank test section.

In 13 of the 14 small tank tests, waves were run in burst durations of 20 seconds. The 20-second burst duration prevented wave energy reflected from the slope from being re-reflected from the wave blade while it was still in motion. Such a condition would cause two distinct wave trains to travel toward the embankment, making it more difficult to measure and describe the incident wave height. The 20-second burst duration

Table 1. Riprap and overlay stability test conditions and results.[1]

Test designation	Stone Type	Stone W (lb)	Coverage	No. of stones per ft^2	C.F.	$H_{D=0}$	N_s^*	W_{50} (lb)	$H_{D=t}$	N_s	Remarks
Small tank											
S-1	11-mm bluestone (quarrystone)	0.00526	Conventional 2.08 in thick	-----	-----	0.121	2.32				Tests S-1, S-2, and S-3 were basic riprap stability tests without overlay.
S-2	11-mm bluestone (quarrystone)	0.00526	Conventional 1.61 in thick	-----	-----	0.110	2.11				
S-3	11-mm bluestone (quarrystone)	0.00526	Conventional 1.57 in thick	-----	-----	0.110	2.11				
S-4	0.75- to 1-in boulders	0.045	100 pct	139.8	0.586	0.176	1.65				
S-5	1- to 1.5-in boulders	0.096	100 pct	76.0	0.528	0.242	1.76				
S-6	1- to 1.5-in boulders	0.097	67 pct	49.4	0.347	0.220	1.60				
S-7	1.5- to 2-in boulders	0.350	100 pct	32.8	0.541	0.363	1.72				Wave generator run continuously
S-8	1.5- to 2-in boulders	0.360	100 pct	32.8	0.545	0.374	1.75				
S-9	1.5- to 2-in boulders	0.360	67 pct	21.8	0.367	-----	----				
S-10	1.5- to 2-in boulders	0.360	67 pct	21.8	0.367	0.319	1.50				
S-11	1.5- to 2-in quartzite (quarrystone)	0.350	100 pct	24.9	0.411	0.363	1.72				
S-12	1.5- to 2-in quartzite (quarrystone)	0.350	100 pct	26.3	0.434	0.352	1.67				
S-13	1.5- to 2-in quartzite (quarrystone)	0.350	100 pct	26.3	0.434	-----	----				
S-14	1- to 1.5-in limestone (quarrystone)	0.073	Conventional 1.93 in thick	247.0	1.434	0.253	2.02				Overlay with conventional two layer thickness
Large tank											
L-1	quartzite (quarrystone)	77.0	100 pct	0.604	0.420	3.150	2.47	100.0	3.50	2.26	
L-2	quartzite (quarrystone)	203.0	100 pct	0.377	0.430	4.020	2.28	209.0	4.38	2.26	

[1]See page 6 for definitions of symbols.

also allowed the generation of 14 to 15 well-formed waves at a test period of 1.16 seconds.

The wave heights referred to in this report are the significant wave heights of the 14 or 15 well-formed waves in each burst. The significant wave height is the average of the highest one-third of the waves in the burst. This does not necessarily correspond to the significant height in an ocean wave spectrum. Although the wave height was spot checked during the stability tests, the actual wave heights used were obtained from a previous calibration of the wave generator. Both the small and large wave tanks were calibrated for wave heights using absorber beaches at the location of the embankment toe. Therefore, the wave heights used in this report represent the actual incident height at the embankment toe. The wave height for each condition tested was predetermined by calibrating the wave tank and running-in bursts. The location of the wave gages used to spot check the wave height in the small tank is shown in Figure 1.

The basic data in this study consisted of surveys of the embankment taken after each run was completed. A run (series of bursts) continued until the embankment slope had reached equilibrium at a particular wave height; however, never less than 50 waves bursts (about 750 well-formed waves) were considered sufficient to constitute a run. Normally, after 50 bursts the slope became stable, the run was terminated, and a survey was taken. In the small wave tank the wave height was increased in increments of from 0.015 to 0.030 foot (4.6 to 9.1 millimeters) between runs.

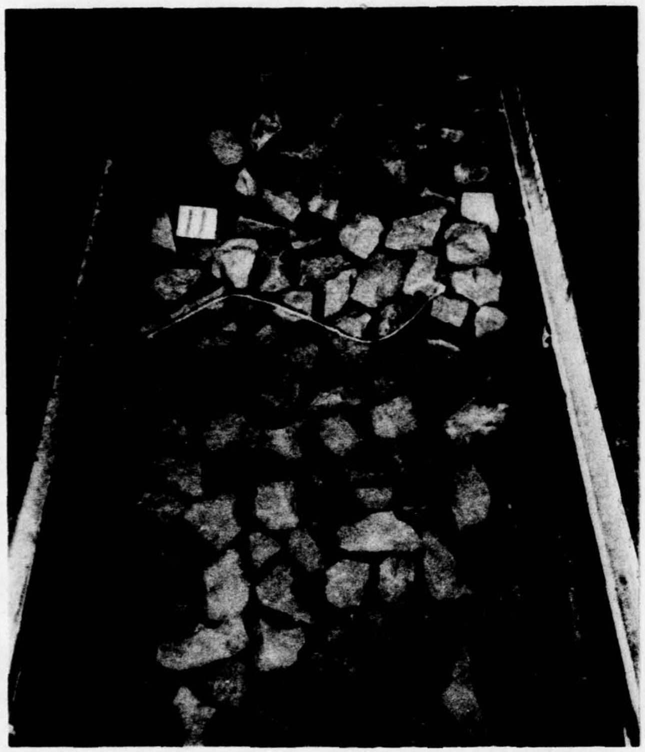
Tests S-1, S-2, and S-3 (Table 1) were used to establish the zero-damage stability of the crushed bluestone which constituted the model riprap layer without overlay, i.e., base condition.

Tests S-4 to S-13 (Table 1) determined the stability characteristics of armor stone overlay. The overlay stones were of two general types, rounded boulders and angular Sioux quartzite quarrystone. Three sizes of boulders were tested and designated 0.75 to 1 inch (19 to 25.4 millimeters), 1 to 1.5 inches (25.4 to 38.1 millimeters), and 1.5 to 2 inches (38.1 to 50.8 millimeters). Both the boulders and the quarrystone had a specific gravity of 2.65. Details on the type of stone, stone size, and weight and coverage fractions for the various tests are given in Table 1.

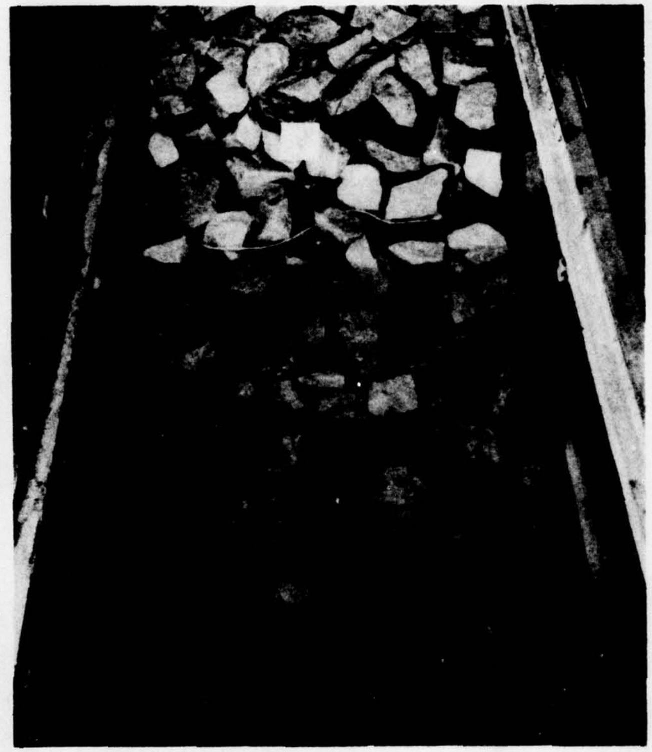
In test S-7 the wave generator was run continuously, as opposed to wave bursts, for a fixed number of waves to see if continuous wave action of less than design height could displace the riprap through the voids in the armor overlay.

Test S-14 was considered to be a conventional overlay test, in that the overlay constituted a layer about two stones in diameter thick, rather than 100- or 67-percent coverage of a single layer of stone on top of the riprap. Kimmswick limestone quarrystone at a designated size of 1 to 1.5 inches was used for this test. The test was used as a reference for comparison with the overlay tests using considerably smaller coverage fractions.

Small tank tests photos of quarrystone and boulders having 100-percent overlay and the boulders having 67-percent overlay are shown in Figures 2, 3, and 4.



Initial condition,
rope denotes stillwater level;
tank 1.5 ft wide;
1.5- to 2-in quartzite



Slope following wave attack of
H_s = 0.375 ft
(minor damage condition);
1.5- to 2-in quartzite

Figure 2. Small wave tank test of 100-percent stone overlay (angular quarrystone).

Initial condition,
rope denotes stillwater level;
tank 1.5 ft wide;
1.5- to 2-in scale boulders

Slope following wave attack of
H_s = 0.525 ft
(minor damage condition);
1.5- to 2-in scale boulders

Figure 3. Small wave tank test of 100-percent stone overlay (boulders).

Initial condition,
rope denotes stillwater level;
tank 1.5 ft wide;
1.5- to 2-in scale boulders

Slope failure, H_s = 0.325 ft;
1.5- to 2-in scale boulders

Figure 4. Small wave tank test of 67-percent stone overlay (boulders).

3. Prototype Tests.

Two prototype overlay tests were conducted in the large wave tank. The riprap underlayer for these two tests was Sioux quartzite quarrystone with a median weight of 3 pounds (1.4 kilograms). Between the riprap underlayer and the core of the embankment there was a crushed stone filter layer 6 inches (15.2 centimeters) thick, using stone with a median diameter of 1.5 inches. The core was compacted bank-run soil. Details of the large wave tank test section are shown in Figure 5.

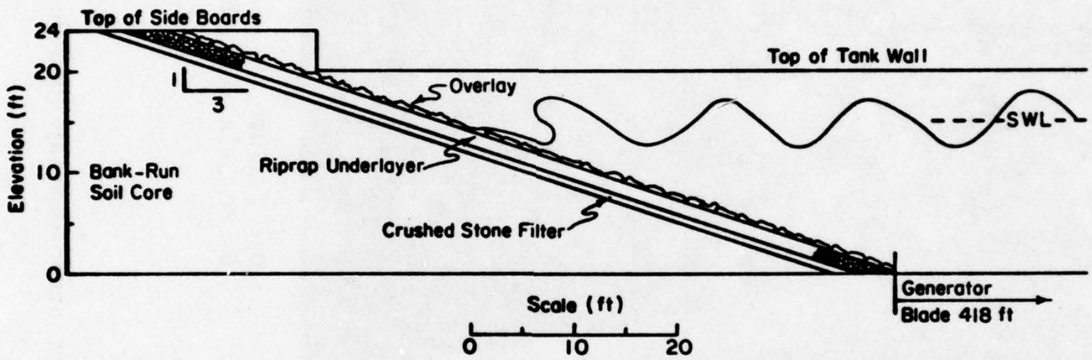


Figure 5. Details of large wave tank test section.

Test L-1 determined the stability of the 100-pound (45 kilograms) median weight Sioux quartzite quarrystone overlay. The largest overlay quarrystone weighed 290 pounds (132 kilograms); the smallest, 20 pounds (9.1 kilograms). This 100-percent overlay was tested for a 3.67-second period wave with a 15-foot stillwater depth in the tank. Wave heights ranged from 2.8 to 3.8 feet (0.85 to 1.16 meters). A total of 100 wave bursts with 19 waves per burst were run at each height.

Test L-2 evaluated the stability of a 100-percent overlay of Sioux quartzite quarrystone with a median weight of 209 pounds (95 kilograms). The largest overlay quarrystone weighed 675 pounds (306 kilograms); the smallest, 100 pounds. The water depth was 15 feet and a 4.2-second wave period was tested. Wave heights ranged from 3.2 to 4.4 feet (0.98 to 1.34 meters). A total of 100 wave bursts with 14 waves per burst were run at each height. In the large tank the wave height was increased in increments of from 0.3 to 0.4 foot (9.1 to 12.2 centimeters) between runs.

The large wave tank testing at prototype scale was done to determine what corrections were needed for the small-scale test results to make the small tank test representative of prototype conditions; i.e., determine scale effects. Large tank test photos of the 100-percent quarrystone overlay (W_{50} = 100 pounds) are shown in Figure 6.

III. ANALYSIS OF TEST RESULTS

Changes in the embankment profile were observed from the survey data collected after each run. These changes were converted to a volumetric

Slope construction

Slope failure, $H_s = 3.85$

Initial condition

Figure 6. Prototype tests of 100-percent stone overlay (quarrystone) coverage.

change relative to the initial survey taken before any waves were run. The volumetric changes were a measure of the damage to the embankment, and were plotted versus wave height to estimate the zero-damage wave height. As each test progressed there was usually some wave height at which an abrupt increase in the damage occurred. The wave height that preceded this abrupt change in the rate of damage is considered the zero-damage wave height, $H_{D=0}$. The damage that occurred to the embankment before reaching the zero-damage wave height was usually insignificant, and resulted from the slight movement of a few stones which were in unstable positions at the completion of construction of the embankment. As an additional aid to establishing the zero-damage wave height, extensive written records of visual observations during the testing were made. Normally, the visual method of estimating the zero-damage wave height was used to confirm the survey data method, and there was always close agreement between the two.

Stability of riprap is often measured as the stability number, N_s. The stability number, which is a dimensionless zero-damage wave height, can be used to compute a stable armor unit weight (Hudson, 1958), and is defined by:

$$N_s = \frac{H_{D=0}}{\left(\frac{W_{50}}{w_r}\right)^{1/3} (S_r - 1)} , \qquad (2)$$

where W_{50} is the median weight of the stone (pounds), w_r is the unit weight of the stone (pounds per cubic foot), and $S_r = w_r/w_w$; w_w is the unit weight of the local water. Since the weight distributions for the overlay stone used in the small tank tests were not obtained, it was convenient to define a stability number based on average weight. The average weight stability number is given by:

$$N_s^* = \frac{H_{D=0}}{\left(\frac{W}{w_r}\right)^{1/3} (S_r - 1)} , \qquad (3)$$

where W is the average weight of the stone (pounds). N_s^* was useful for making comparisons between various small-scale tests and between small-scale and prototype tests since the average weight was known for all tests. However, it should be noted that N_s^* is not the stability number normally used.

An embankment with a stone overlay was considered a failure when enough of the stone overlay and riprap underlayer had been removed by wave forces, leaving the filter layer clearly exposed. An embankment without a stone overlay was considered a failure when enough riprap had been removed to clearly reveal the filter.

Another useful definition is the tolerable-damage wave height, $H_{D=t}$, which is the largest wave height that does not remove either filter or riprap material through voids of the stone overlay. A wave height of $H_{D=t}$ will move some overlay stones around but the damage is considered tolerable

because the revetment integrity is maintained. Both $H_{D=0}$ and $H_{D=t}$ are given in terms of the significant wave height at the toe of the embankment. Test conditions and results are presented in Table 1.

1. Discussion.

The tests show that riprap stability is greatly improved when a stone overlay is used. It might be suspected that when the waves become large enough to move the overlay stone, open areas would be created in the overlay through which the underlying riprap could be removed by the wave action. Small open areas did appear between the overlay stone when the wave height reached the zero-damage level and gradually enlarged with increasing wave height. These open areas started near the stillwater level and slowly migrated upslope, sometimes extending to the upper limit of the active wave action. The open areas developed through the general shifting around of the overlay stones, which tended to pack more tightly just below the stillwater level, rather than by the actual removal of overlay stones. Even as the open areas enlarged, the exposed riprap underlayer had little tendency to be removed and remained sheltered from the wave action by the overlay stones until the waves approached the failure wave height. Near the failure wave height, riprap was removed from the open areas, undermining the stability of the adjacent overlay stones which shifted around and further enlarge the area; at times, the overlay stones were also removed. The riprap and overlay stones once removed from the open area were deposited by the wave in the zone just below the stillwater level.

In test S-7, where the wave generator was run continuously rather than in 20-second bursts, the prolonged attack of high waves failed to remove the riprap through the overlay stones. There was no observable displacement of overlay stones at a wave height of 0.363 foot (11.1 centimeters), the wave height increment just below the estimated zero-damage wave height. These observations were supported by time-lapse movies of the riprap motion. Table 2 shows the following three comparisons of the stone overlay stability, using the average weight stability number, N_s^*:

(a) The stability of 100- to 67-percent overlay coverage;

(b) the stability of a rounded to angular stone overlay; and

(c) the stability of a quarrystone overlay for small-scale and prototype tests.

Since the number of tests involved in the comparisons (Table 2) were small, the results are regarded only as trends. The last comparison indicates surprisingly large-scale effects, but the differences in stability may be partly due to differences in the gradation and shape of the overlay. The gradation of the stone overlay used in the prototype tests (Table 3) is considered wider than the stone overlay used in the small-scale tests, although the gradation of the overlays in the small-scale tests were not documented. The quarrystone overlay used in the prototype

Table 2. Comparison summary of stone overlay stability.

Comparison	Tank	No. of tests	Avg. N_s^*	Results
Boulders,				
100-pct coverage	small	4	1.72	A 10-pct reduction in N_s^* for 67-pct overlay.
to				
67-pct coverage	small	2	1.55	
Boulders and quarrystone,				
100-pct boulders	small	4	1.72	Boulders and quarrystone 100-pct overlays have equal stability.
to				
100-pct quarrystone	small	2	1.70	
Quarrystone,				
100-pct quarrystone	large	2	2.38	Small-scale N_s^* must be increased by 40 pct to correspond to prototype tests results.
to				
100-pct quarrystone	small	2	1.70	

Table 3. Comparison of stone gradations.

Comparison	W_{max}	W_{min}
Ahrens, 1975	$4W_{50}$	$0.125W_{50}$
This study (prototype tests)	$3.1W_{50}$	$0.4W_{50}$

tests appeared to be more blocky than the quarrystone overlay in the small-scale tests (compare Figs. 2 and 6). However, the scale effects shown in Table 2 are consistent with the findings of Thomsen, Wohlt, and Harrison (1972) who found sizable scale effects on riprap stability in small-scale model tests.

To determine how general the findings of this study are, it would be valuable to compare the results with a study of conventional riprap stability. Ahrens (1975) is a convenient and useful comparison since the stone gradations are similar and a wide range of wave and slope conditions were tested at prototype scale. A comparison of stone gradations used in Ahrens (1975) and this study is given in Table 3. The tests by Ahrens were conducted in the CERC large wave tank and most used a riprap layer between 1.5- and 2-median-stone diameters thick. A riprap layer thickness between these diameters is considered a conventional two-layer riprap. Revetment slopes of 1 on 2.5, 1 on 3.5, and 1 on 5 were tested at prototype scale for wave periods between 2.8 and 11.3 seconds. The conditions and corresponding results which most closely matched these stone overlay tests are:

Slope	Period (s)	No. of tests	Avg. N_s
1 on 2.5	4.2	4	1.99
1 on 3.5	4.2	4	2.42

Estimated N_s for 1 on 3 slope = 2.21

The interpolated N_s = 2.21 for a conventional 1 on 3 riprap is approximately equal to the N_s = 2.26 for a 1 on 3, 100-percent stone overlay armor (Table 1). The similarity of the zero-damage stability between the stone overlay and conventional riprap suggests that the stability equation for conventional riprap design, developed by Ahrens and McCartney (1975), can be used with equation (2) to estimate stable stone overlay weights. This equation is:

$$N_s = 1.46 \; (\cot \theta)^{2/9} \quad , \tag{4}$$

where θ is the angle between the embankment face and the horizontal. Equation (4) is intended for design use, and was made conservative enough to account for the worst wave conditions and scatter in the test results. No allowance is made in equation (4) for uncertainty in predicting the design significant wave height. If equation (4) is solved for cot θ = 3, the stability number is 1.86, which is about 20 percent lower than the large wave tank overlay test results. The lower stability number is indicative of the conservatism included in equation (4).

The limit of tolerable damage is the maximum wave height a structure can withstand without some loss of structural integrity. The ratio of tolerable-damage wave height to zero-damage wave height is a measure of

the structure's reserve stability. The reserve stability of the 100-percent overlay armor is estimated by the following large wave tank results:

Test	$H_{D=0}$ (ft)	$H_{D=t}$ (ft)	Reserve stability
L-1	3.15	3.50	1.11
L-2	4.02	4.38	1.09
		Average	1.10

Reserve stability of conventional two-layer riprap was determined in the large wave tank by Ahrens (1975) as follows:

Slope	No. of tests considered	Avg. reserve stability
1 on 2.5	9	1.16
1 on 3.5	13	1.22

Interpolation of this data for a 1 on 3 slope yields a reserve stability of 1.19; *i.e.*, the riprap still protects the embankment from damage for a wave about 20 percent higher than the zero-damage wave height. By comparison, the 100-percent overlay has only one-half as much (10 percent) reserve stability as the conventional two layers of armor.

The number of stones per unit area necessary to have the 100-percent stone overlay condition (all stones touching) is different for boulders and quartzite quarrystone because of stone shape. The coverage fraction (Table 1) shows 100-percent coverage of boulders having a coverage fraction between 0.53 to 0.59; the 100-percent coverage for quarrystone (including prototype and small-scale tests) has a coverage fraction between 0.41 and 0.43. This means that to obtain 100-percent coverage about 30 percent more of the rounded boulders would be required per unit area than the blocky quarrystone.

The variation of coverage fraction between boulders and conventional shape riprap used in these tests is consistent with an evaluation of riprap layer thickness reported by Hudson (1958). This change of coverage fraction with layer thickness for various stone shapes is shown in Figure 7.

The variation of coverage fraction in Figure 7 can be used to make comparative cost estimates of alternative stone types for revetment design. An example of the use of Figure 7 is given in Appendix B.

2. Conclusions.

The general conclusion of these prototype and small-scale wave tank tests is that a one-layer stone overlay greatly improves riprap stability

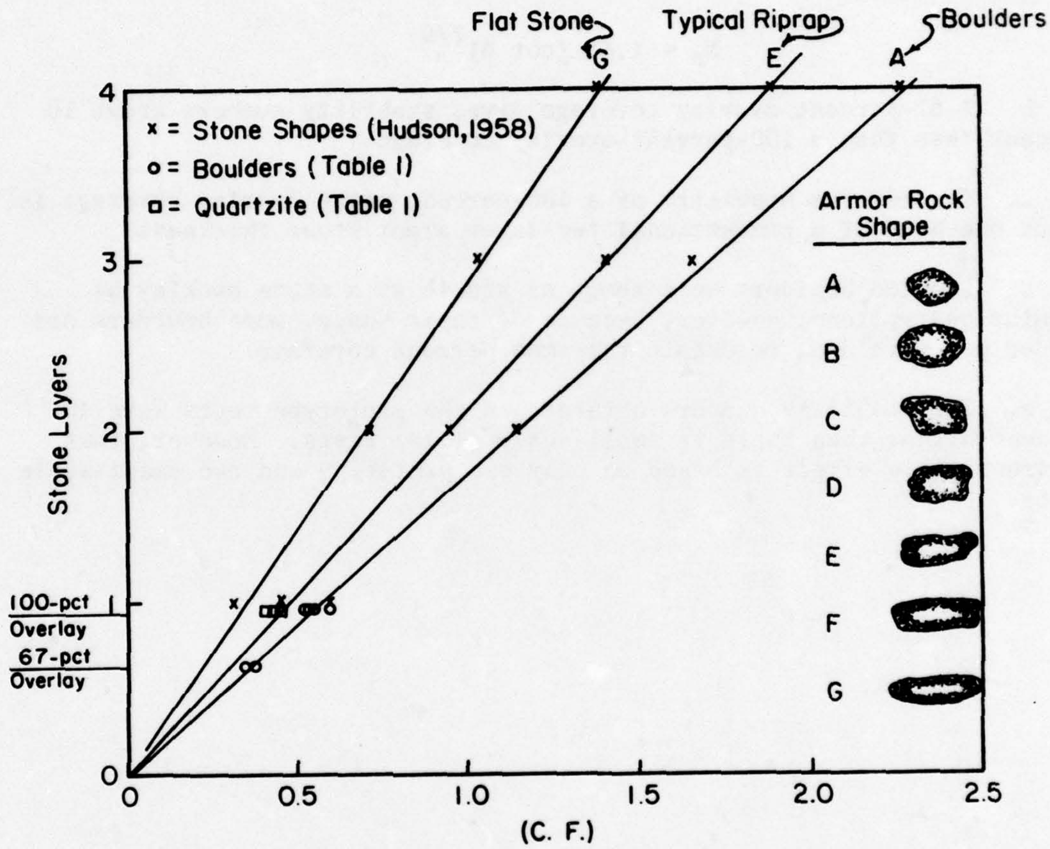


Figure 7. Stone coverage fraction for overlay coverage and armor shape.

and can be useful for upgrading smaller riprap slope protections. The following specific conclusions were also made:

a. A 100-percent stone overlay coverage gives about the same stability number, N_s, as a conventional two-layer armor stone thickness. Overlay stone stability numbers for a 100-percent overlay can be computed from the formula:

$$N_s = 1.46 \ (\cot \theta)^{2/9} \quad .$$

b. A 67-percent overlay coverage gives stability numbers about 10 percent less than a 100-percent overlay coverage.

c. The reserve stability of a 100-percent stone overlay coverage is about one-half of a conventional two-layer armor stone thickness.

d. Rounded boulders were about as stable as a stone overlay as angular quarrystone; however, because of their shape, more boulders are needed per unit area to obtain the same percent coverage.

e. The stability numbers obtained in the prototype tests were 40 percent higher than those in small-scale (1:10) tests. However, this apparent scale effect is based on only two prototype and two small-scale tests.

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APPENDIX A

CASE STUDY OF OAHE RESERVOIR

Oahe Dam is a 9,300-foot-long (2.83 kilometers), 230-foot-high (70.1 meters) earthfill dam located on the Missouri River in central South Dakota (Fig. A-1).

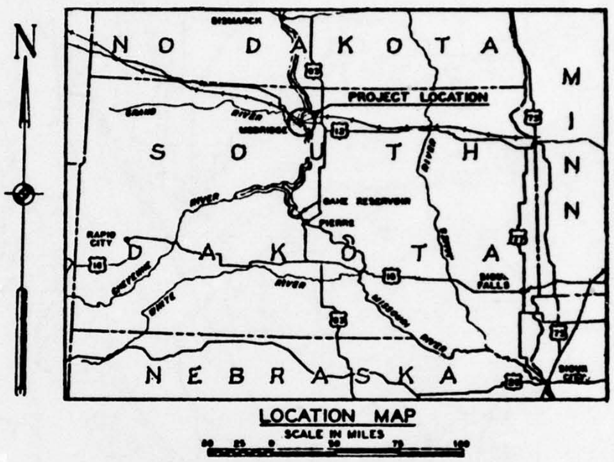


Figure A-1. Oahe Dam.

The reservoir behind Oahe Dam, filled in 1962, is about 190 miles (360 kilometers) long and 5 to 10 miles (8 to 16.1 kilometers) wide. About 100 miles (161 kilometers) upstream from the dam, a railroad crosses the reservoir near the town of Mobridge, South Dakota. The railroad crossing is made by a 4,500-foot-long (1.37 kilometers) earth embankment reveted by riprap and a 2,000-foot-long (610 meters) bridge. Figure A-2 is a project map showing the railroad embankment and the reservoir segment near the embankment.

The embankment core is dredge-fill material with 1 on 3 side slopes. Both upstream and downstream slopes are reveted with a 6-inch-thick bed of gravel, 6-inch-thick layer of spalls, and a 24-inch-thick (61 centimeters) layer of riprap armor. This riprap revetment has a top elevation of 1,627 feet (496 meters) mean sea level (MSL) on both upstream and downstream sides. The toe elevation of the upstream side is 1,543 feet (470 meters) MSL and 1,588 feet (484 meters) MSL on the downstream side. The reservoir operating levels (U.S. Army Engineer District, Omaha, 1969) in feet above MSL are:

Maximum operating pool	1,620
Maximum normal operating pool	1,617
Minimum flood control pool	1,607.5
Limit of drawdown	1,540

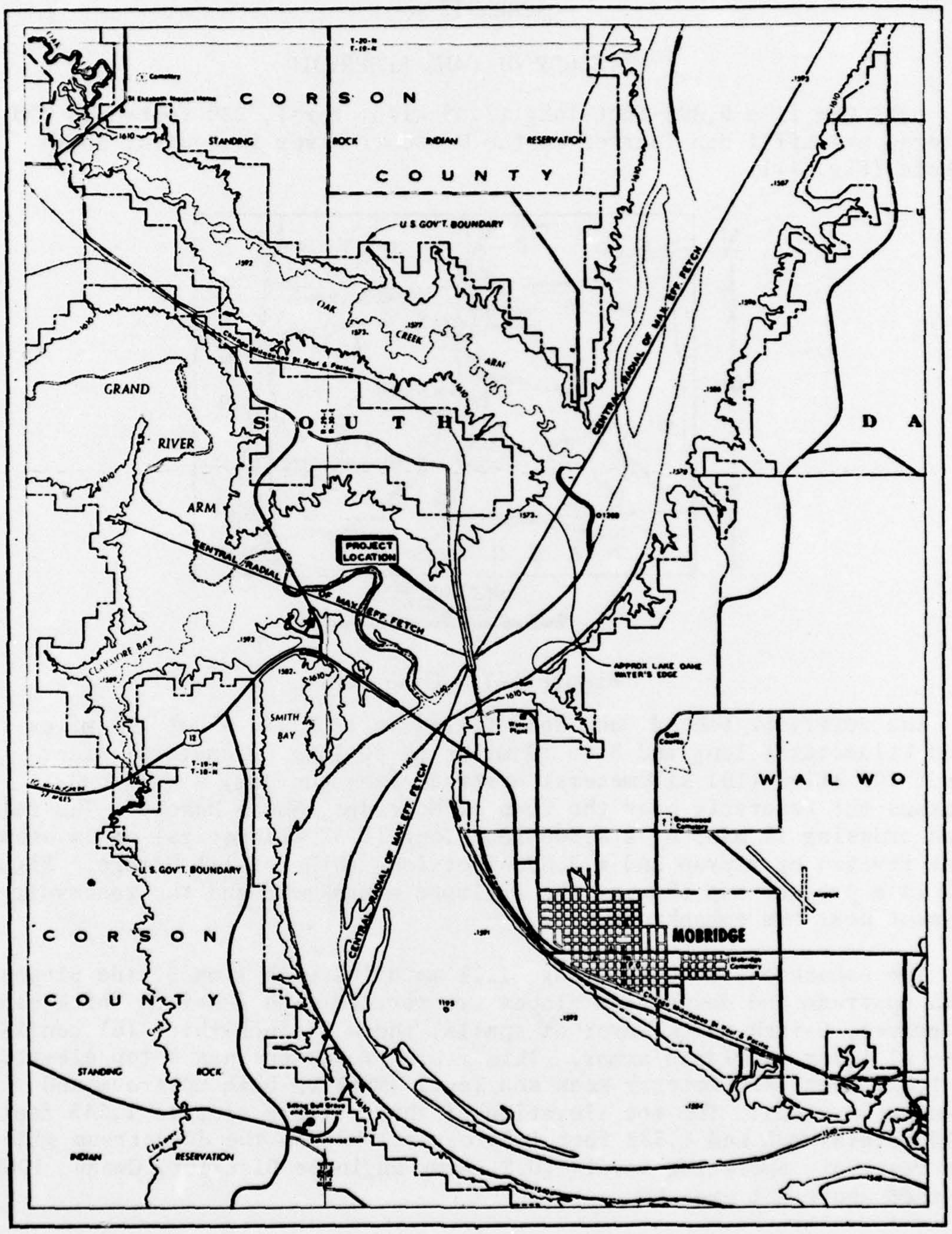


Figure A-2. Project map, railroad embankment in Lake Oahe.

The specified armor stone gradation called for riprap with a medium weight of 125 pounds (57 kilograms). However, the actual median riprap weight in place ranged between 10 and 75 pounds (4.5 and 34 kilograms), which was considerably less than specified. The specified riprap gradation and actual riprap gradations are shown in Figure A-3.

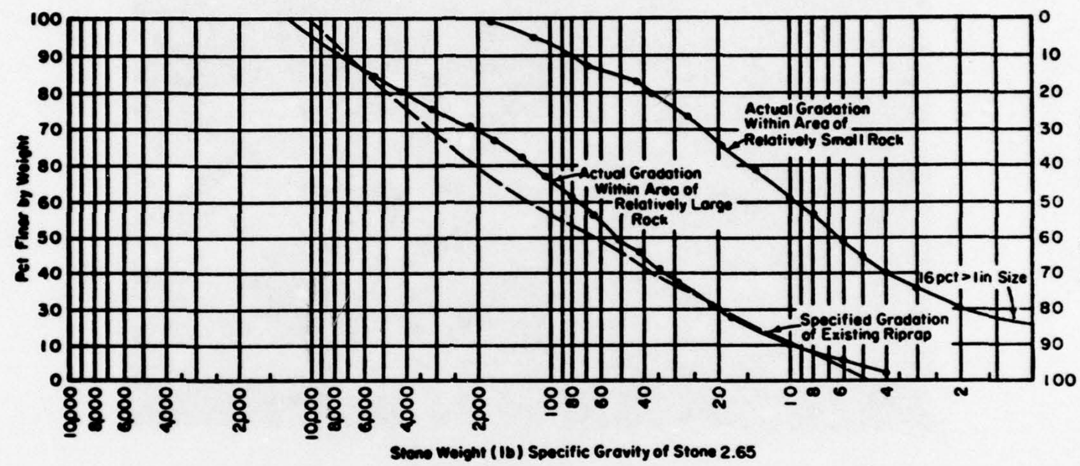


Figure A-3. Riprap gradation for railroad embankment (U.S. Army Engineer District, Omaha, 1969).

From 1962 to 1967 the reservoir pool level fluctuated between elevations of 1,555 and 1,597 feet (474 and 487 meters). The first riprap failure was noticed on the upstream face in June 1967. Damage consisted of a 2,700-foot-long (823 meters) wave-cut notch similar to that shown in the photos in Figure A-4. This damage was attributed to storm waves from the northeast on 30 April and 1 May 1967 during which the Mobridge Weather Bureau Station reported gusts to 70 miles (113 kilometers) per hour.

More riprap damage occurred on the upstream slope during 1968 at a higher reservoir pool level. This damage notch was not as extensive as the 1967 damage although it was about 2,500 feet (762 meters) long. Typical sections of the damaged riprap revetment are shown in Figure A-5.

U.S. Army Engineer District, Omaha (1969) estimated a 5-foot (1.52 meters) wave caused the 1967 and 1968 slope damage. This estimate was based on the wave-cut notch and position of driftwood on the slope. Further analysis by the District concluded that the embankment would be exposed during a 100-year period to 250 waves with a height of 5 feet or higher. This 5-foot wave height was selected as the design wave for the permanent revetment repair. Assuming the 5-foot height approximately represents the 1-percent exceedance wave height of a Rayleigh distribution, the significant height is 3.3 feet (1.01 meters). The 1-percent exceedance wave height is the height exceeded by 1-percent of the waves. Therefore, the design significant wave height, H_s, is 3.3 feet.

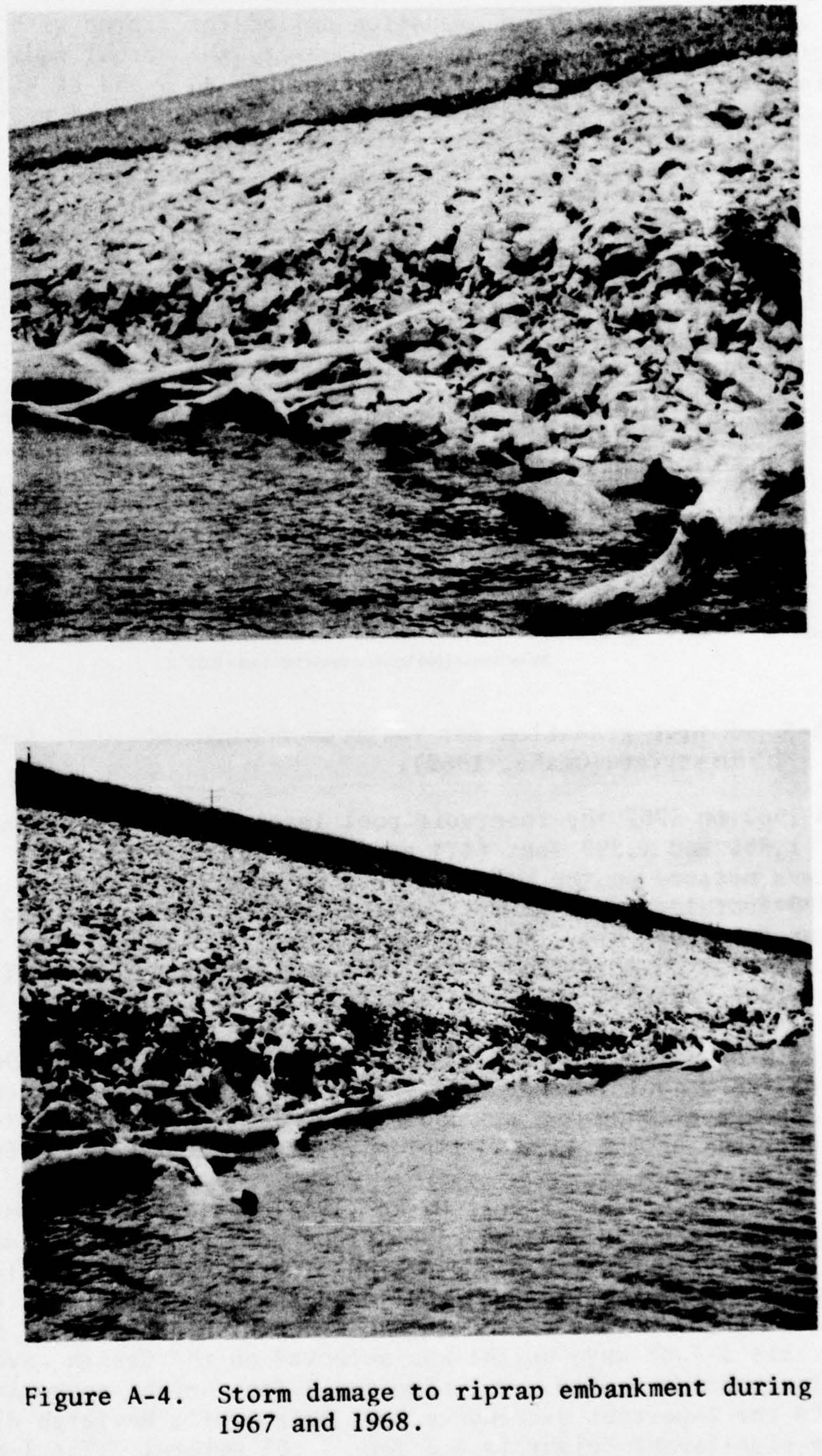


Figure A-4. Storm damage to riprap embankment during 1967 and 1968.

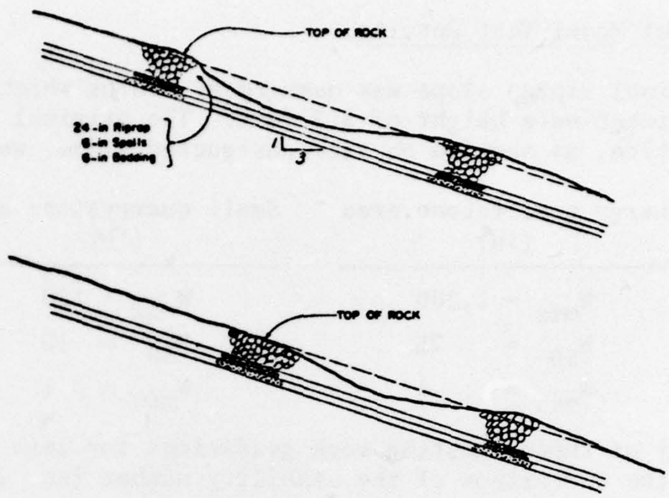


Figure A-5. Typical damaged riprap sections (U.S. Army Engineer District, Omaha, 1969).

The upstream embankment slope is exposed to an effective fetch across the reservoir of about 1.4 miles (2.25 kilometers); the downstream slope is exposed to a 1.5-mile (2.41 kilometers) effective fetch. Using the deepwater wave forecasting curves in U.S. Army, Corps of Engineers, Coastal Engineering Research Center (1975), a H_s = 3.3 feet can be generated by a wind of about 55 miles (89 kilometers) per hour blowing over a 1.5-mile fetch. A 3.6-second wave period is associated with this 3.3-foot-high significant wave height. Thus, a storm with gusts of 70 miles per hour over a 1.5-mile fetch is capable of generating some waves as high as 5 feet, the designated design height.

Both the 1967 and 1968 damage was temporarily repaired by dumping quarrystone into the notches from a barge. While these emergency repairs provided temporary protection, a permanent solution to the inadequate riprap stability problem was needed. U.S. Army Engineer District, Omaha (1969) evaluated the following methods of upgrading the riprap stability: (a) Overlaying with wire mesh or chain-link fence anchored in place to hold the stone and prevent movement; (b) placing grout by various methods into the voids between the existing stone to bind smaller stone together into either larger or continuous solid units that are more wave resistant; (c) overlaying with grouted preplaced mesh-enclosed coarse aggregate as suggested by Milwaukee railroad officials; (d) overlaying with manufactured concrete armor units such as tetrapods or tribars; and (e) overlaying with quarrystone or boulders. Method (e) was selected as the cost-effective plan with a good chance of success.

To assure success of the stone overlay design, the U.S. Army Engineer District, Omaha requested a series of wave tank tests be performed at the Coastal Engineering Research Center (CERC). These tests were intended to determine stability of various stone overlay coverages for both quarrystone and boulders.

Application of Model Test Results.

The original riprap slope was damaged by storms which had an estimated significant wave height of 3.3 feet. The original 1 on 3 slope riprap gradation, as sampled on the constructed slope, was:

Large quarrystone area (lb)	Small quarrystone area (lb)
$W_{max} = 1{,}200$	$W_{max} = 180$
$W_{50} = 75$	$W_{50} = 10$
$W_{min} = 4$	$W_{min} = < 1$

The stability of these existing rock gradations for zero damage is computed using the definition of the stability number (eq. 2) rearranged to yield:

$$W_{50} = \frac{w_r\, H^3}{N_s{}^3 - (S_r - 1)^3} ,$$

where

W_{50} = median stone weight (pounds)

w_r = unit weight of stone (165 pounds per cubic foot)

H = significant wave height (feet)

$N_s = 1.46\ (\cot\theta)^{2/9} = 1.86$ (Ahrens and McCartney, 1975).

θ = angle of structure slope with the horizontal ($\cot\theta = 3$)

S_r = armor specific gravity, $= \frac{165}{62.4} = 2.64$

Using this equation, the zero-damage wave heights for the damaged railroad embankment slope follow:

Area	W_{50} (lb)	Zero-damage wave height (ft)
Small stone (riprap)	10	1.2
Large stone (riprap)	75	2.4

This analysis of the original riprap shows it was considerably smaller than the size required to be stable against the design waves. Using this same equation for the design significant wave height of 3.3 feet, W_{50} = 209 pounds is required for stability. This computed stable stone overlay weight of 209 pounds was increased arbitrarily to 600 pounds (272 kilograms) to incorporate a factor of safety into the repair design and because stone up to this weight was available at about the same unit

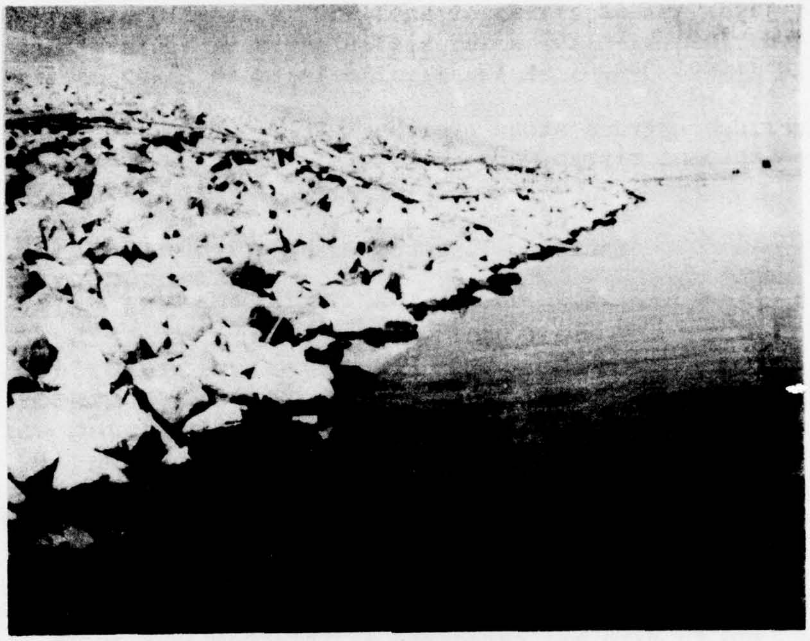
cost as smaller overlay stone. Since W_{50} is proportional to H^3, the 600-pound median weight stone overlay should be stable for waves with a significant height up to 4.7 feet (1.43 meters).

The Mobridge upgrade stone overlay specifications for repair of the railroad embankment riprap were:

Weight of individual stones (lb)	Pct smaller by weight
1,200	100
600	35 to 50
300	6 to 10

Overlay coverage was to be 50 tons per 1,000 square feet above water and 65 tons per 1,000 square feet below water (U.S. Army Engineer District, Omaha, 1971).

The stone overlay on the railroad embankment was constructed in 1971 (Fig. A-6), and to date has performed satisfactorily without the need for additional repair work.



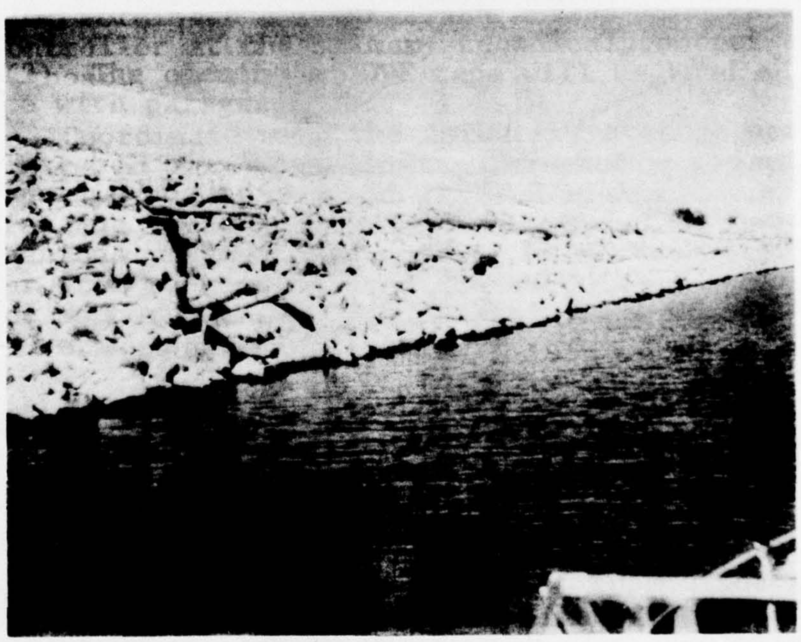


Figure A-6. Damaged revetment repair by stone overlay.

APPENDIX B

EXAMPLE PROBLEM

This appendix illustrates how the findings of this study can be used to calculate the weight of stone and amount of stone required for a stable overlay-type revetment. Guidance is provided on how this information can be used to develop a cost-effective design.

*****************************EXAMPLE PROBLEM*****************************

GIVEN:

(a) 1 on 3 slope existing riprap revetment damaged by H_s = 5 feet, T = 4 seconds.

(b) Stability number is $N_s = 1.86$ $[N_s = 1.46 \ (\cot \theta)^{2/9}]$ for a 1 on 3 slope riprap revetment.

FIND:

(a) A 100-percent stone overlay size (W_{50}) necessary to stabilize slope against wave H_s = 5 feet, T = 4 seconds.

(b) Stone weight per unit area for overlay of (1) boulders, (2) overlay of conventional riprap-shaped quarrystone, and (3) conventional two-layer overlay.

Stable quarrystone weight for:

$$H_s = 5 \text{ feet}$$

$$S_r = \frac{165}{62.4} = 2.64$$

$$w_r = 165 \text{ pounds per cubic foot}$$

$$W_{50} = H^3 \, w_r \left[(N_s{}^3 \ (S_r - 1)^3 \right]$$

$$W_{50} = (5^3) \ (165)/(1.86^3) \ (2.64 - 1)^3$$

$$W_{50} = 727 \text{ pounds.}$$

$$\text{Use } W_{50} = 730 \text{ pounds.}$$

From Figure 7, stone coverage fraction (C.F.) for boulders and riprap:

(a) C.F. = 0.55 for boulders (100-percent coverage).

(b) C.F. = 0.43 for riprap (100-percent coverage, "E" stone shape).

ANSWERS:

Weight of boulders needed per 1,000 square feet of revetment surface area:

$$\frac{\text{weight}}{1{,}000\ \text{ft}^2} = C(1{,}000) = \text{C.F.}\left(\frac{W_{50}}{w_r}\right)^{1/3} w_r(1{,}000/2{,}000)$$

$$= 0.55\ (730/165)^{1/3}(165)\left(\frac{1{,}000}{2{,}000}\right)$$

$$= 74.5 \text{ tons per } 1{,}000 \text{ square feet.}$$

Weight of quarrystone riprap needed per 1,000 square feet of revetment surface area:

$$\frac{\text{weight}}{1{,}000\ \text{ft}^2} = 0.43\ (730/165)^{1/3}(165)\left(\frac{1{,}000}{2{,}000}\right)$$

$$= 58.2 \text{ tons per } 1{,}000 \text{ square feet.}$$

Conventional two layers of overlay have about the same zero-damage wave height stability as the 100-percent overlay. Therefore, a conventional two-layer overlay W_{50} = 730 pounds. From Figure 7:

C.F. = 0.95 quarrystone riprap for two layers

C.F. = 1.15 boulders for two layers

$$\text{Quarrystone riprap weight} = 0.95\left(\frac{730}{165}\right)^{1/3} 165\left(\frac{1{,}000}{2{,}000}\right)$$

$$= 128.7 \text{ tons per } 1{,}000 \text{ square feet.}$$

$$\text{Boulder weight} = 1.15\left(\frac{730}{165}\right)^{1/3} 165\left(\frac{1{,}000}{2{,}000}\right)$$

$$= 155.8 \text{ tons per } 1{,}000 \text{ square feet.}$$

SUMMARY:

Overlay	Overlay W_{50} for zero-damage H_s = 5 ft (lb)	Overlay (tons per 1,000 ft^2)
100-percent boulders	730	74.5
100-percent quarrystone riprap	730	58.2
Boulders	730	155.8
Quarrystone riprap, two layers	730	128.7

If unit prices for boulders and conventional quarrystone riprap are available, an economic comparison can be made.

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Overlay of large, placed quarrystone and boulders to increase riprap stability / by Bruce L. McCartney and John P. Ahrens. – Fort Belvoir, Va. : U.S. Coastal Engineering Research Center, 1976.
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Bibliography : p. 23.
This report describes the wave tank tests and field performance of a single layer of large armor stone used as a protective overlay on underdesigned riprap. The resistance of the overlay to wave attack was determined by small-scale model and prototype-scale wave tank tests at CERC. Design information on a stone overlay concept used to repair a damaged riprap revetment on Oahe Reservoir, South Dakota, is also included.
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